TECH BRIEF

Geotechnical Design of Ductile Iron Piles in Compression

Ductile Iron Piles (DIPs) support allowable axial compression loads ranging from 25 tons to more than 100 tons. The compression design capacity of DIPs depends on characteristics of the selected pile materials and the prevailing geotechnical conditions. Design capacities are determined by calculating the structural capacity (allowable strength) of the pile material and estimating the geotechnical capacity (soil resistance) and selecting the smaller of these two values. Structural design approaches are described in detail in Tech Brief – Structural Design of Ductile Iron Piles in Compression.

This document describes the design methodology to evaluate the allowable geotechnical capacity of Ductile Iron Piles to resist compression loads. The design methodology includes consideration of both side friction along the pile as well as end-bearing at the pile tip. Design examples and load test examples highlighting performance are provided in the Appendix.

BACKGROUND

The Ductile Iron Pile system is a low vibration, driven pile system. The system is manufactured in Europe by centrifugal or spin casting of spheroidal graphite cast iron composed of an iron-carbon-silicon alloy. The material exhibits high impact resistance, ultimate strength and elastic limit. The alloy also retains more typical characteristics of cast iron including high compressive strength, fatigue resistance, and corrosion resistance. Modular pile sections are manufactured in 5 meter (16.4 ft) lengths with a Plug & Drive connection mechanism. Each section has a tapered socket with an internal shoulder for full engagement at one end and a tapered spigot at the other end. This system rapidly connects to form a pile of virtually any length without the effort of dedicated field welding or splicing.

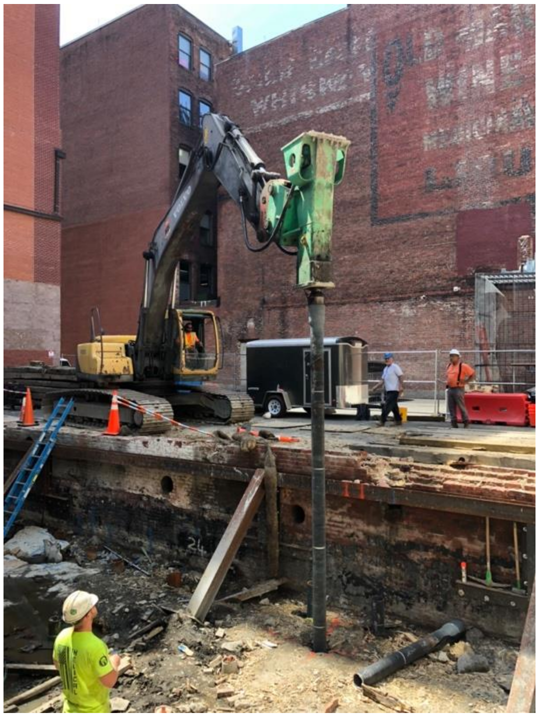


Figure 1: Ductile Iron Pile Installation

As pictured in Figure 1, the modular system uses an excavator-mounted, high-frequency hydraulic hammer fitted with a special drive adapter that rapidly advances the pile into the ground using a combination of excavator crowd force and the percussive (ramming) energy from the hammer.

Ductile Iron Piles are manufactured in three diameters: 98 mm (3.89 in), 118 mm (4.65 in) and 170 mm (6.70 in) with different pile wall thicknesses that range from 6.0 mm (0.24 in) to 13.0 mm (0.51 in). The strength and stiffness properties for design include a yield stress of 320 MPa (46.4 ksi) and an elastic modulus value of 170,000 MPa (24,600 ksi). Additional material properties and dimensions for the different pile sizes documented by the manufacturer (Tiroler Rohre, GmbH) are available in the **Ductile Iron Pile Spec Sheet**.

INSTALLATION

Ductile Iron Piles are installed using either a dry method (non-grouted exterior) or a wet method (exterior grouted). Figure 2 provides pile details for each of the different methods. Although not always the case, the dry method is most commonly installed to terminate on a very hard layer (i.e. rock or glacial till) to develop the majority of geotechnical capacity in end-bearing. Conversely, a wet installation method using exterior grouted piles is commonly used to maximize geotechnical frictional resistance through grout-to-ground bonding within a competent soil layer.

For the **dry installation method**, a drive shoe is placed on the end of the first modular pile section. The modular pile section is driven in the ground. The tapered end of the next pile section is inserted into the bell of the first section and the pile is advanced further. This process is repeated until the pile reaches the competent bearing stratum and the penetration rate slows due to the increased resistance to penetration. The pile is driven until the depth at which "set" is achieved. "Set" is defined as a particular penetration rate (time over advancement). A typical set criterion used for the pile system is 1 inch or less of movement in 50 seconds when installed with a hammer of recommended energy for the specific pile size. The project set criterion is most commonly established based on site-specific drive rates in combination with a successful full-scale load test program. After achieving set, the pile is cut to the appropriate elevation and the interior is grouted for added strength and corrosion resistance. If required by the design, a high strength center bar is then immediately inserted into the grout.

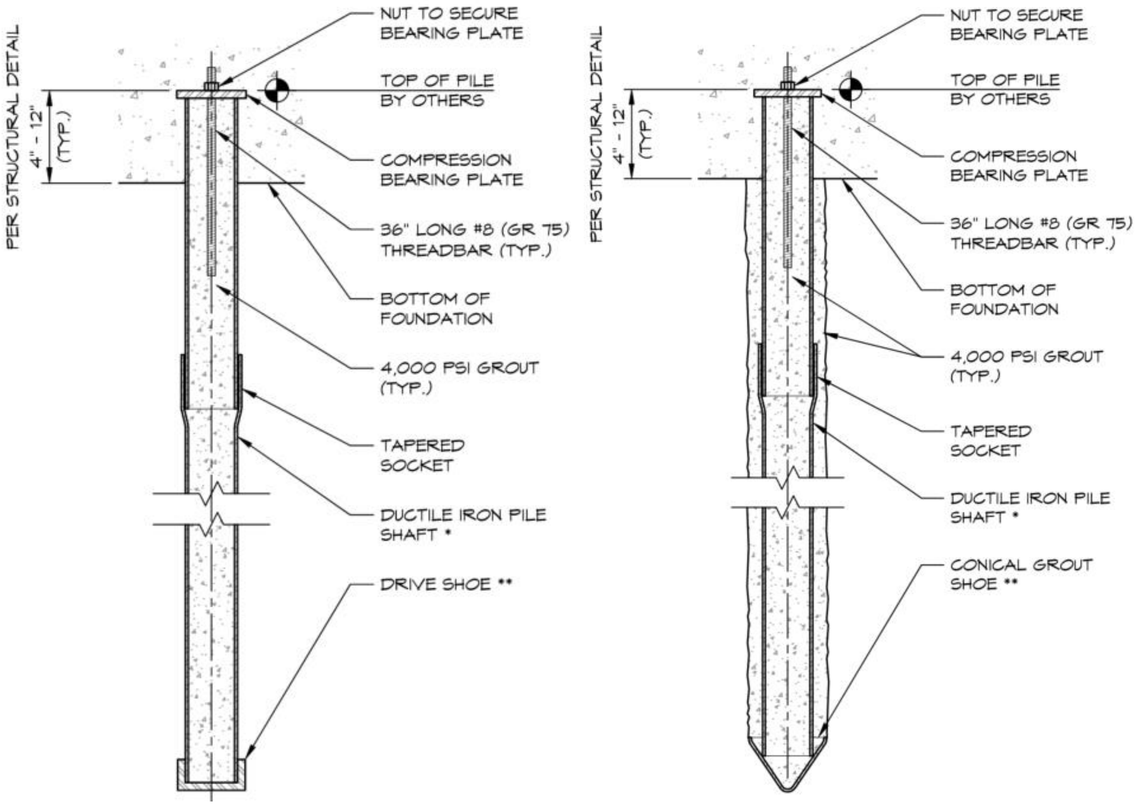


Figure 2: Example Details of Ductile Iron Piles in Compression – a) Non-grouted (Exterior) and b) Grouted (Exterior)

For the **wet installation method** (exterior-grouted), piles are installed using an oversized grout shoe at the pile tip that creates an annular space around the pile while driving. Cement grout is continuously pumped through the center of the pile and out of the grout shoe to fill the annular space that is created while advancing the pile. The pile is driven to the design length within competent soil strata to develop sufficient bond length for frictional capacity. After reaching the final length, the pile is cut to the appropriate elevation. If required, a high strength center bar is immediately lowered into the wet grout. In cases where high tension or lateral resistance is required, piles may still be installed with a wet installation method but advanced to a competent stratum to achieve "set."

More information on the installation process can be found in the DuroTerra brochure.

Design

Geotechnical design of Ductile Iron Piles for compression resistance considers the frictional resistance along the pile shaft ($Q_{s\text{-}ult}$) within competent soil stratigraphy as well as the end-bearing resistance at the pile tip ($Q_{eb\text{-}ult}$). The allowable geotechnical capacity (Q_{all}) is determined based on the combination of the shaft friction and end-bearing capacity divided by the factor of safety (FS) as shown in Equation 1.

$$Q_{all} = \frac{Q_{s-ult}+Q_{eb-ult}}{FS} \qquad \text{Eq. 1}$$

A factor of safety of 2.0 is often used when full-scale load testing is performed. Higher values of 2.5 or 3.0 may be more appropriate when load testing is not performed, and the capacity is determined by analysis only.

Frictional Capacity Design

The geotechnical frictional capacity of the Ductile Iron Pile ($Q_{s\text{-}ult}$) considers a cylindrical shearing surface as a function of the pile length and outermost pile diameter. For piles installed using the dry method (non-grouted exterior), the outermost diameter simply refers to the outer diameter of the ductile iron pile material (i.e. Series 170/9.0 has an outer diameter of 170 mm). The increased diameter of the bell is typically neglected. For piles installed using the wet method (grouted exterior), the outermost diameter that defines the perimeter cylindrical surface is based on the diameter of the grouting shoe. For instance, a Series 170/9.0 pile size installed with a 270 mm (10.6 in) grout shoe is designed with a diameter of about 10.6 inches because the exterior grout column defines the shearing surface.

Multiple analysis approaches are commonly used in the industry to evaluate pile capacity depending on the pile type and the soil conditions. Geotechnical capacity for Ductile Iron Piles can be estimated based on a simplified grout-to-ground bond stress approach (Sabatini et al 2005) often used for micropiles bonding in soil or an effective stress method like the Beta Method (Hannigan et al 2016).

Bond Stress Method

For exterior grouted piles, the grout forms a grout-to-ground bond along the length of the pile. This bond engages the soil during loading to provide shaft friction along the pile. The grout-to-ground bond is typically estimated as a bond capacity value (α_{bond}) that is a function of the soil type and stiffness or density. Example bond values for Type B micropiles (pressure grouted) that are most similar to values observed during Ductile Iron Pile load testing are shown in Table 1 (Sabatini et al 2005). Note that while the tabulated bond values are independent of depth or effective stress, experience shows that reduced grout-to-ground values should be considered in situations with shallow friction piles and/or with high groundwater scenarios to account for low effective stress levels.

Table 1: Grout-to-Ground Bond Values (after Sabatini et al 2005)

Soil Description	Grout-to-Ground Bond Ultimate Strength, psi (kPa)
Silt & Clay (some sand) (soft, medium plastic)	5 – 14 (35 – 95)
Silt & Clay (some sand) (stiff, dense to very dense)	10 – 27.5 (70 – 190)
Sand (some silt) (fine, loose-medium dense	10 – 27.5 (70 – 190)
Sand (some silt, gravel) (fine-coarse, med. - very dense)	17.5 – 52 (120 – 360)
Gravel (some sand) (medium – very dense)	17.5 – 52 (120 – 360)
Glacial Till (silt, sand, gravel) (medium – very dense, cemented)	14 – 45 (95 – 310)

The ultimate shaft resistance for an exterior grouted Ductile Iron Pile within a uniform soil deposit is estimated as shown in Equation 2:

$$Q_{s-ult} = \pi D L_b \alpha_{bond} \qquad \text{Eq. 2}$$

where D is the outermost diameter of the Ductile Iron Pile (including exterior grout column) and L_b is the bond length. The shaft resistance should refer only to soils where competent resistance is anticipated. For instance, very soft soils, organic soils or some undocumented fills may be inappropriate for generating adequate geotechnical resistance for design and should be neglected from the calculation. In layered soil conditions, the resistance within each competent soil layer corresponding to the length and bond value is calculated and combined to determine the total allowable shaft resistance. Additionally, it is important that variations in strain rates between different soil types be considered when selecting appropriate bond values for design. An example of a frictional capacity design using the bond stress method is provided in Example 1 of the Appendix.

Effective Stress Method

For piles installed using the dry method (non-grouted exterior) or as an alternative to the Bond Stress Method, an Effective Stress Method can be used to estimate the shaft resistance along the pile. This method applies the Beta Method (Hannigan et al 2016) where the frictional resistance is a function of the horizontal effective stress around the pile as well as the interface friction between the pile surface and the soil. Similar to the Bond Stress Method, only competent soil layers should be considered as contributing to the available shaft resistance.

The ultimate shaft resistance for a friction Ductile Iron Pile using the Effective Stress Method is estimated as shown in Equation 3:

$$Q_{s-ult} = \pi d L \sigma'_v K_s (\tan \delta) \quad \text{Eq. 3}$$

where d is the outer diameter of the Ductile Iron Pile material, L is the pile length within a particular layer, K_s is the coefficient of lateral earth pressure, σ'_v is the average vertical effective stress along the pile shaft within the layer and δ is the interface friction angle between the pile surface and the surrounding soil. The expression $K_s \tan \delta$ is often referred to as the Bjerrum-Burland beta coefficient. Values for the beta coefficient range significantly in the literature depending on the ground conditions (soil type, composition, density, etc) as well as the pile type and pile surface characteristics. Displacement piles (i.e. driven piles) typically have higher values than other types of piles (i.e. drilled) due to the volumetric expansion and displacement that occurs during the driving process particularly in densifiable conditions.

A general range of beta coefficient values for various soil conditions vary from 0.15 to 0.90 after Fellenius (Hannigan 2016). Other approaches like those developed by Nordlund indicate higher values of the equivalent beta coefficient values depending on the soil friction angle and amount of volumetric displacement that occurs. Ductile Iron Pile load test experience based on tension tests as well as compression tests instrumented with strain gauges suggest the beta coefficient values are often on the higher end of the range, particularly for the non-cohesive soils due to the displacement and corresponding densification that occurs in the surrounding soils during driving combined with the high interface friction between the pile surface and the soil due to the roughened Ductile Iron Pile surface that occurs from the manufacturing process. Unlike a smooth steel pipe pile, the surface of a Ductile Iron Pile is roughened and more comparable to the undulations on a golf ball. Further, the interface friction value for an exterior grouted Ductile Iron Pile where the grout bonds to the surrounding soil are typically equal to the full soil friction angle. An example of a frictional capacity design using the effective stress method is provided in Example 2 of the Appendix.

End-Bearing Capacity Design

Estimates of geotechnical end-bearing resistance can be developed based on traditional pile capacity analysis methods (i.e. Nordlund Method, etc) (Hanigan et al, 2016). For instance, the following equation estimates the tip resistance capacity ($Q_{eb\text{-}ult}$) in an end-bearing pile.

$$Q_{eb-ult} = \alpha_t N'_q \sigma'_p \left(\frac{\pi D_{tip}^2}{4} \right) \quad \text{Eq. 4}$$

where α_t is a dimensionless coefficient that depends on the pile depth-width relationship and the soil friction angle, N'_q is a bearing capacity factor, σ'_p is the vertical effective stress at the tip of the pile and D_{tip} is the diameter at the pile tip. The vertical effective stress is limited to a maximum value of 3,130 psf (150 kPa). As presented in Hanigan et al (2016), Figures 3 and 4 provide charts of the pile depth-width relationship as well as the bearing capacity factor versus soil friction angle, respectively.

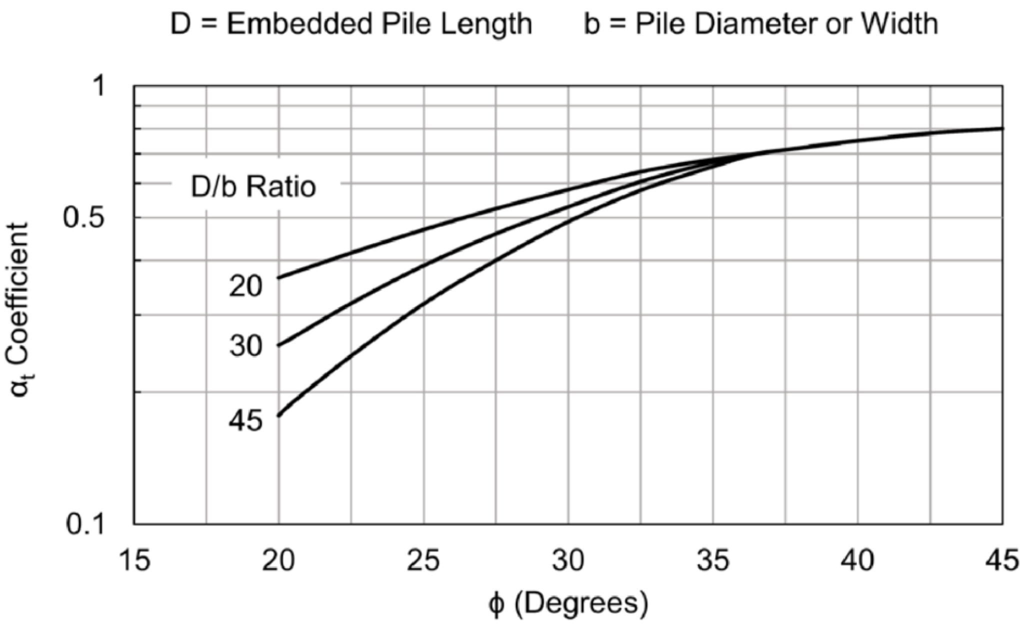


Figure 3. Chart of α_t coefficient versus friction angle

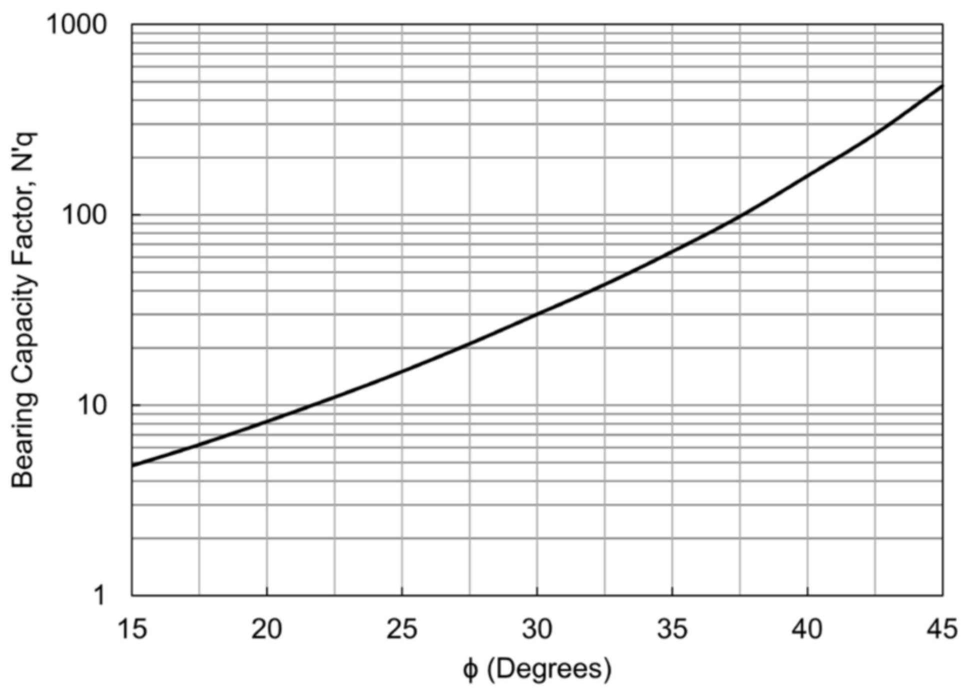


Figure 4. Chart of bearing capacity factor versus soil friction angle

While traditional analysis approaches described above can be used to estimate the end-bearing capacity of Ductile Iron Piles terminating in a range of conditions, Ductile Iron Piles designed to develop significant capacity in end-bearing resistance require the pile be terminated on or driven into very dense glacial soils or rock. These competent soils conditions are commonly characterized by SPT N-values that exceed 50 or 100 blows in 2 inches or less or where auger refusal was encountered during exploratory drilling. It is most common for piles developing end-bearing capacity in these types of conditions to be driven to achieve a project specific "set" criteria (typically on the order of 1-inch or less of movement in 50 seconds or longer) to achieve the geotechnical capacity. Since these traditional geotechnical analyses methods often produce a wide range of capacities due to a high degree of variability and sensitivity of the input values, confirmation of end-bearing capacity is most often based on empirical results associated with full-scale load testing. This is particularly important when the end-bearing conditions are highly competent materials. An example of an end-bearing capacity design for a pile terminating on dense soils without achieving "set" is provided in Example 2 of the Appendix. An example of a pile driven to "set" to achieve capacity is provided in Example 3.

Other Geotechnical Design Considerations

In addition to geotechnical capacity determination, deep foundation design often must consider other factors, which may include estimated pile deflection, group pile settlement/capacity, negative shaft friction or "down drag" and other items. These design factors are beyond the scope of this Tech Brief but can be evaluated in more detail using a range of industry publications and references. Further information can be provided by the Ductile Iron Pile representative.

CONCLUSIONS

Ductile Iron Piles are routinely used for support of foundations loaded in compression. Allowable pile capacities typically range from 25 to 120 tons. There are multiple installation approaches that are selected based on site working conditions, soil conditions and design requirements. The design procedures for estimating geotechnical capacity depend on the installation technique as well as the soil conditions. This Tech Brief describes the installation, various design methods and provides example calculations as evidence of the design approaches delivering superior performance of the system.

REFERENCES

Hannigan, P.J., Rausche, F., Likins, G.E., Robinson, B.R., Becker, M.L. (2016). Design and Construction of Driven Foundations. FHWA-NHI-16-009, Geotechnical Engineering Circular (GEC) No. 12 – Volume I. US. Dept. of Transportation, Federal Highway Administration.

Sabatini, P.J., Tanyu, B., Armour, T., Groneck, P., Keeley, J. (2005). Micropile Design and Construction. FHWA-NHI-05-039, Reference Manual for NHI Course 132078. US. Dept. of Transportation, Federal Highway Administration.

Tiroler Rohre Gmbh (TRM). (2014). "Piling Systems for Deep Foundations." October, 2014.

EXAMPLE 1 - Shaft Frictional Capacity using Bond Stress Method

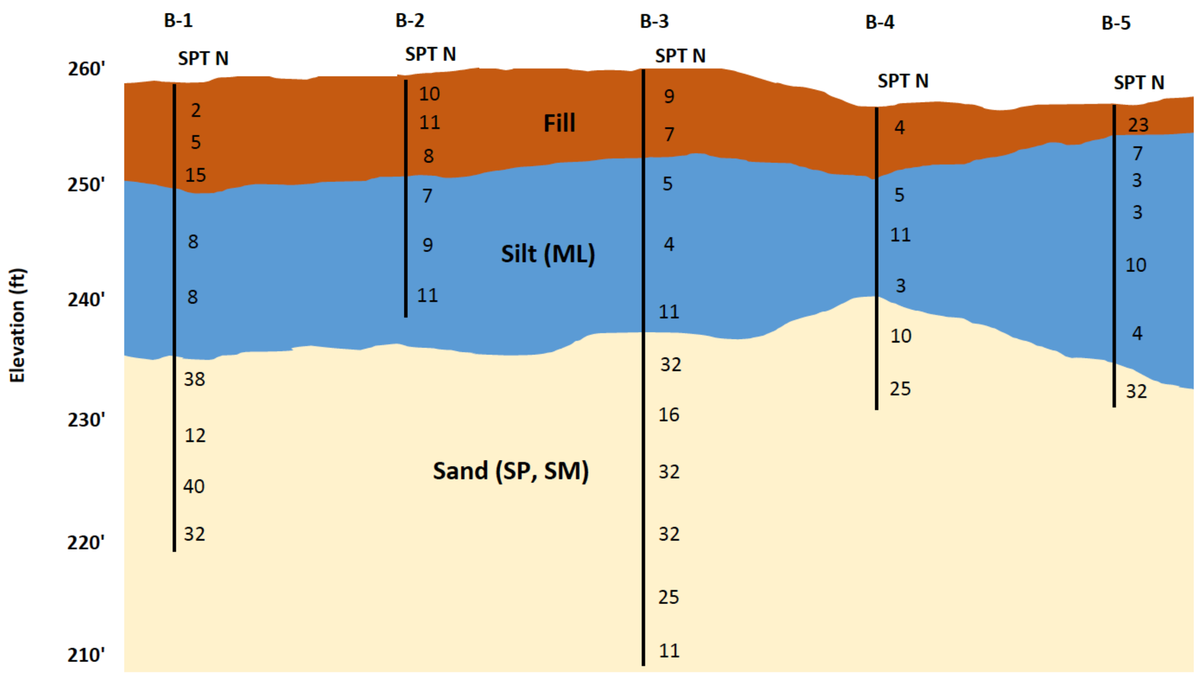
<u>Ductile Iron Pile (DIP) Design Loading</u>

Ductile Iron Pile (DIP) Design Loading	100 kips

<u>Material Selection:</u>

Ductile Iron Pile (DIP) Type:	118 / 7.5

<u>Soil Model:</u>



<u>Frictional Pile Capacity - Bond Stress Method</u>

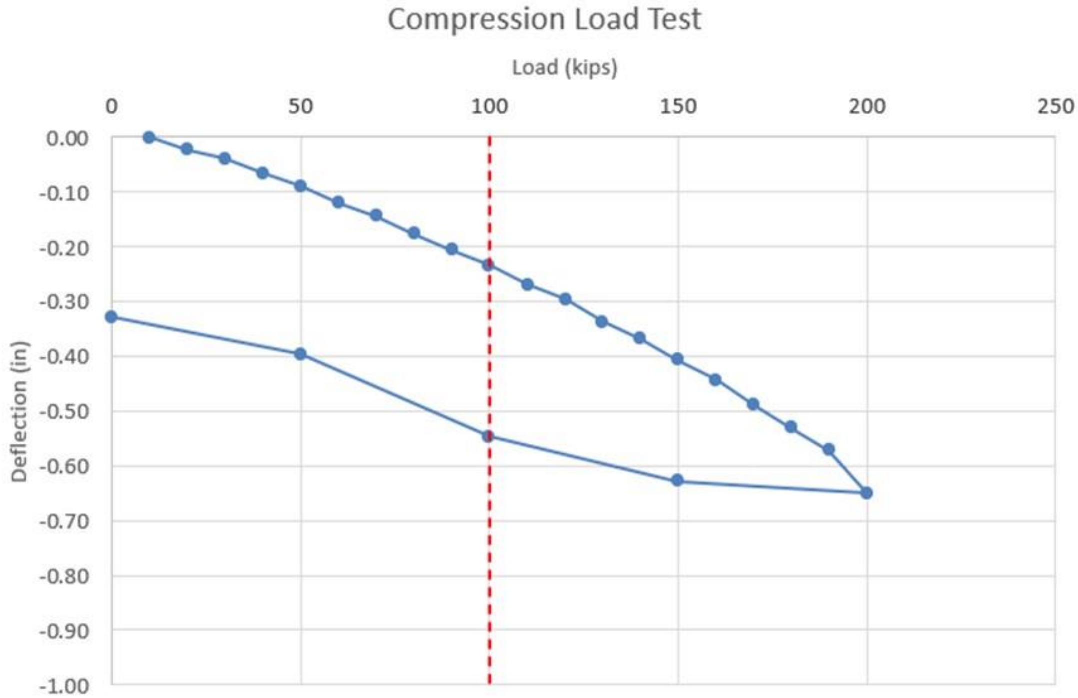
Exterior-grouted Ductile Iron Piles designed as friction piles. Check the geotechnical capacity for an estimated bond length and bond values.

Geotechnical Design of Ductile Iron Piles in Compression

Ductile Iron Pile grout shoe selection:		220 conical		
Grout shoe diameter:	$d_{grout\ shoe}$	8.7	in	
Selected diameter (assumed grout column diameter):	$d_{grout\ shoe\text{-}design}$	8.7	in	
Assumed bond length *(Zone 1)* :	$L_{b\text{-}zone\ 1}$	8	ft	
Ultimate bond stress *(Zone 1)* :	$\alpha_{bond\text{-}zone\ 1}$	0	psi	*(Neglect undocumented fill)*
Assumed bond length *(Zone 2)* :	$L_{b\text{-}zone\ 2}$	15	ft	
Ultimate bond stress *(Zone 2)* :	$\alpha_{bond\text{-}zone\ 2}$	6	psi	*(Bond in silt)*
Assumed bond length *(Zone 3)* :	$L_{b\text{-}zone\ 3}$	20	ft	
Ultimate bond stress *(Zone 3)* :	$\alpha_{bond\text{-}zone\ 3}$	28	psi	*(Bond in sand)*
Total Bond Length:	$L_{b\text{-}total}$	43	ft	

Ultimate Ductile Iron Pile Frictional Capacity:				
$Q_{s-ult} = \pi D L_b \alpha_{bond}$	$Q_{s\text{-}ult}$	**213**	**kips**	
Factor of Safety:	**FS**	**2.0**		*(with load testing)*
Allowable Ductile Iron Pile Frictional Capacity:	$Q_{s\text{-}all}$	**107**	**kips**	
Check compressive capacity versus demand:		**ok**		

Results of a full-scale load test performed at the site on a 49-ft long, exterior-grouted Series 118/9.0 Ductile Iron Pile installed with a 220 mm grout shoe.



EXAMPLE 2 - Shaft Frictional Capacity using Effective Stress Method and End-Bearing Capacity

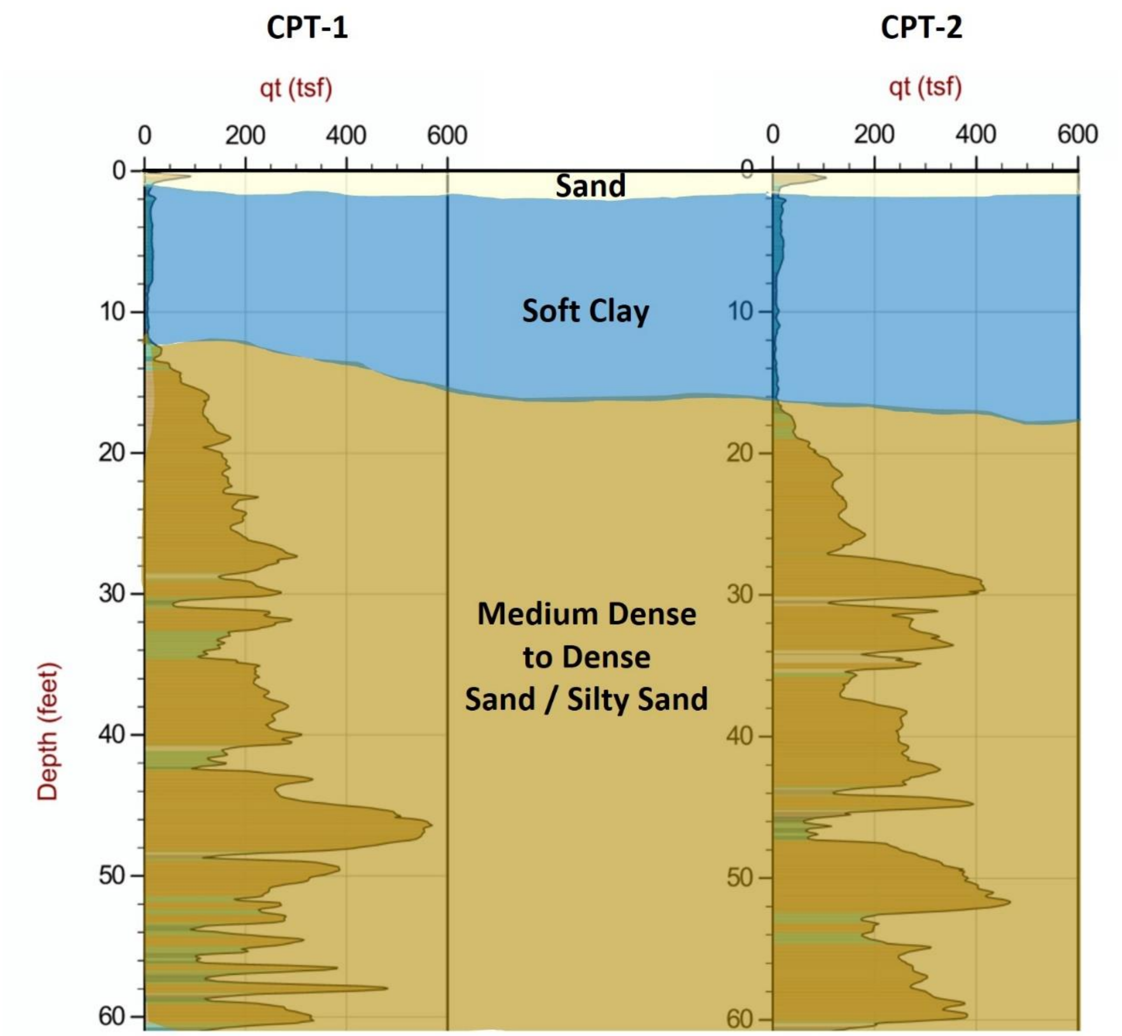
Ductile Iron Pile (DIP) Design Loading

Vertical (compression) load: 60 kips

Material Selection:

Ductile Iron Pile (DIP) Type: 170 / 9.0

Soil Model:



Geotechnical Design of Ductile Iron Piles in Compression

Frictional Pile Capacity - Effective Stress Method

Ductile Iron Piles will be designed to develop capacity in both friction and end-bearing for this project, check the estimated length and resulting capacity.

Pile-to-Soil Frictional Interaction

Pile Diameter (in)	6.69
Groundwater depth (ft)	8

$$Q_{s-ult} = \pi d L K_S \sigma_v' (\tan\delta)$$

Layer Description	Thickness, T (ft)	Total Unit Weight, γ_{tot} (pcf)	Interface Friction Angle, δ (deg)	Lateral Stress Ratio, (K_s)	Include Layer in Shaft Resistance	Depth to Top of Layer, D_{Top} (ft)	Depth to Bottom of Layer, D_{bot} (ft)	Horizontal Effective Stress, $\sigma'_{h\text{-}top}$ (psf)	Horizontal Effective Stress, $\sigma'_{h\text{-}bot}$ (psf)	Ultimate Shaft Capacity, $Q_{s\text{-}ult}$ (kips)
Clay	7	110	16	1.0	Yes	0	7	0	770	1
Clay	8	110	16	1.0	Yes	7	15	770	1213	4
Sand	5	120	22	1.0	Yes	15	20	1213	1501	5
Sand	10	120	27	1.0	Yes	20	30	1501	2077	16
Sand	10	120	27	1.0	Yes	30	40	2077	2653	21
Sand	10	120	27	1.0	Yes	40	50	2653	3229	26

Total Length: 50 ft

Total Ultimate Shaft Capacity (kips)	73
Factor of Safety	2.0
Allowable Shaft Capacity (kips)	**36.7**

End-Bearing Pile Capacity

Pile Shoe Diameter *(HD Shoe)*	d_{shoe}	7.8	in
Depth of bottom of pile	D_{tip}	50	ft
Limiting Stress (σ'_v up to max of 3,130 psf)	p_t	3130	psf
Bearing Capacity Factor	N'_q	120	
Dimensionless pile width/depth factor	α_t	0.75	
Factor of safety	FS	2.0	
	$Q_{end\text{-}bearing\text{-}ult}$	**46.7**	**kips**

$$Q_{eb-ult} = \alpha_t N_q' \sigma_p' \left(\frac{\pi D_{tip}^2}{4}\right)$$

Combined Allowable Geotechnical Capacity **Q_{all}** **83.5 kips**

Check compressive capacity versus demand

ok

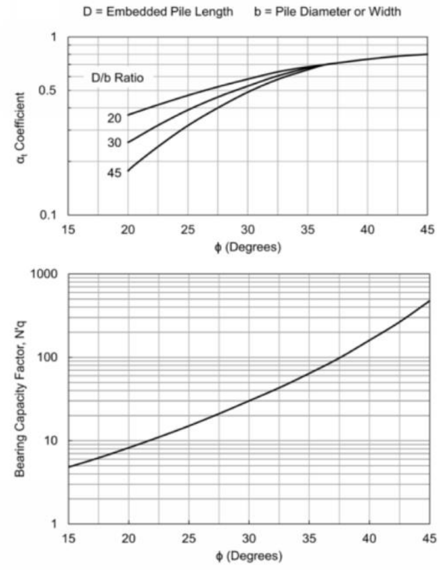
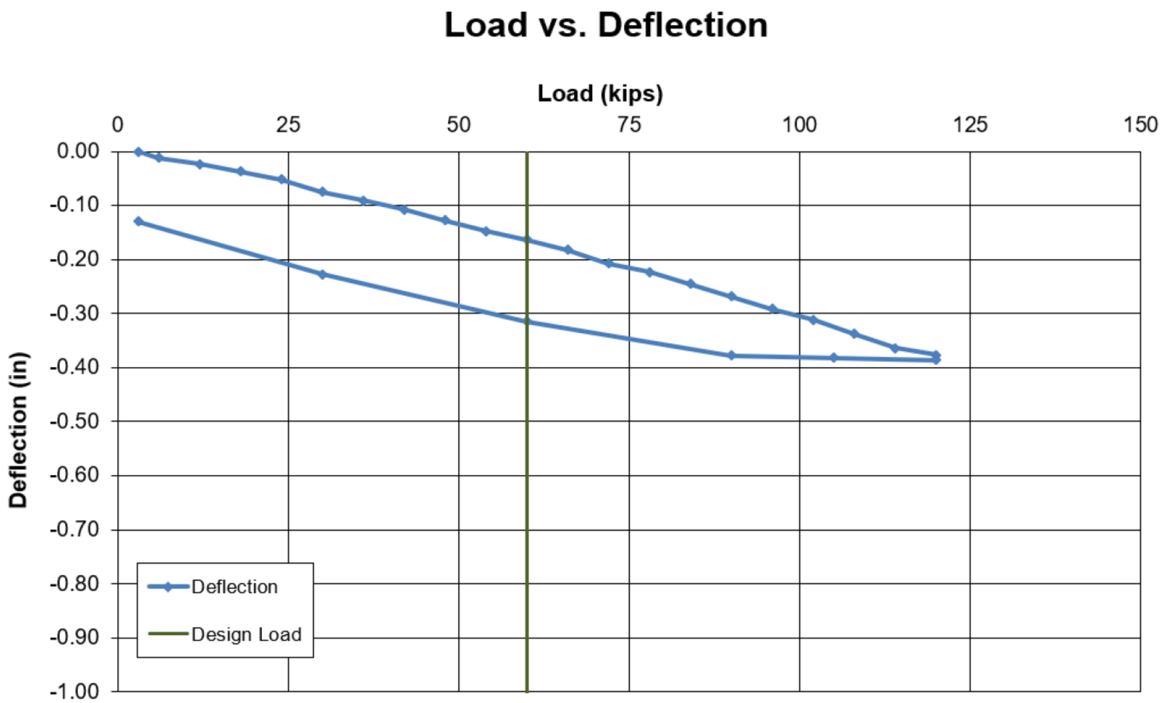


Figure 7-16 Chart for estimating α_t coefficient and bearing capacity factor N'_q (after Bowles 1977).

Results of a full-scale load test performed at the site on a 49-ft long, Series 170/9.0 Ductile Iron Pile installed with a 170 heavy duty (HD) drive shoe.



Geotechnical Design of Ductile Iron Piles in Compression

EXAMPLE 3 - End-Bearing with Set Criteria

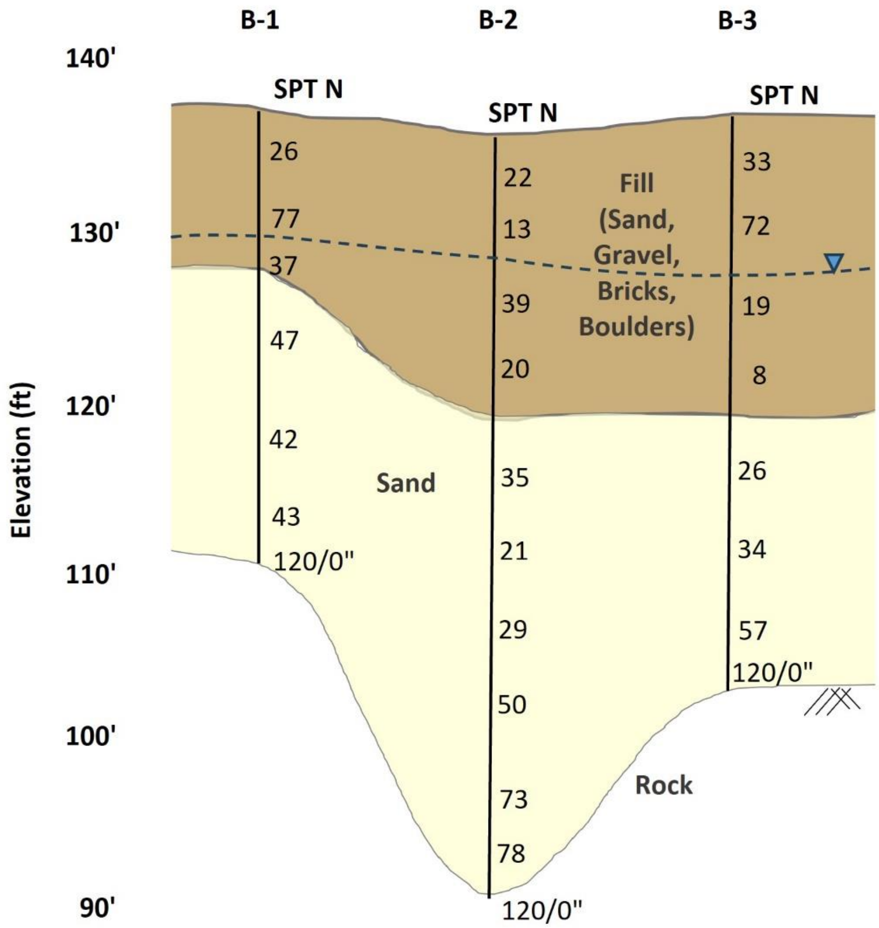
Ductile Iron Pile (DIP) Design Loading

Vertical (compression) load: 150 kips

Material Selection:

Ductile Iron Pile (DIP) Type: 170 / 9.0

Soil Model:



Geotechnical Design of Ductile Iron Piles in Compression

Design Approach and Performance:

Since Ductile Iron Piles will terminate on very dense glacial till or rock by achieving a project specific-set, the design relies on empirical experience from load testing as opposed to theoretical calculations to show the end-bearing capacity. A typical "set" criterion of 1" of movement or less in 50 seconds or more was utilized utilized for the test pile on the project. Compression load test results on the 28-ft long pile terminating on rock by achieving "set" showed a nearly linear and elastic response under loading up to 200%. In addition, data from strain gauges showed only small amounts of load dissipation out of the pile with significant load transferred to the pile tip bearing on rock.

Results of Full Scale Load Test (Load vs. Deflection)